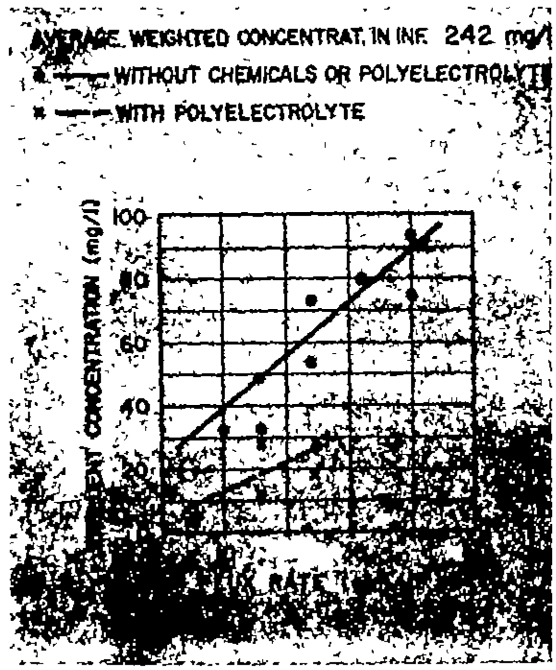


Figure 5
Filtration System Performance
Effluent Suspended Solids Quality

Figure 6
High-Rate Filtration Plant
Flow Diagram

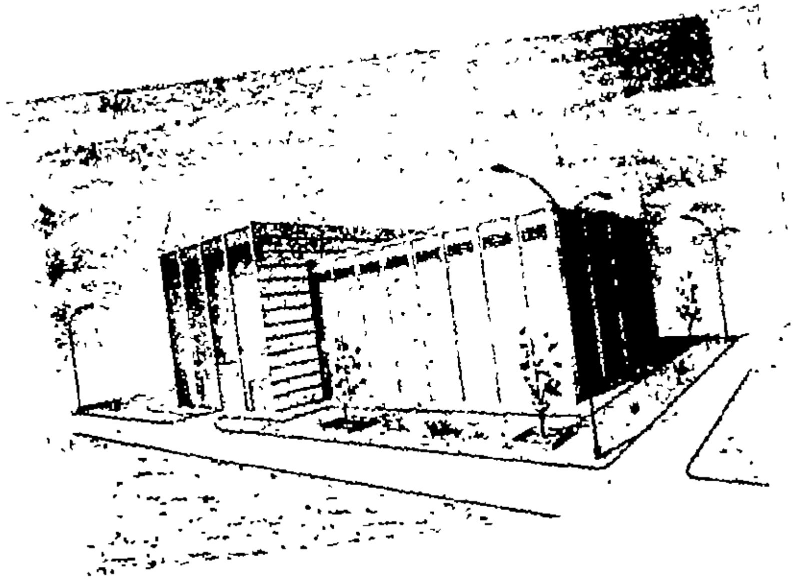


Figure 7
High Rate Filtration Installation

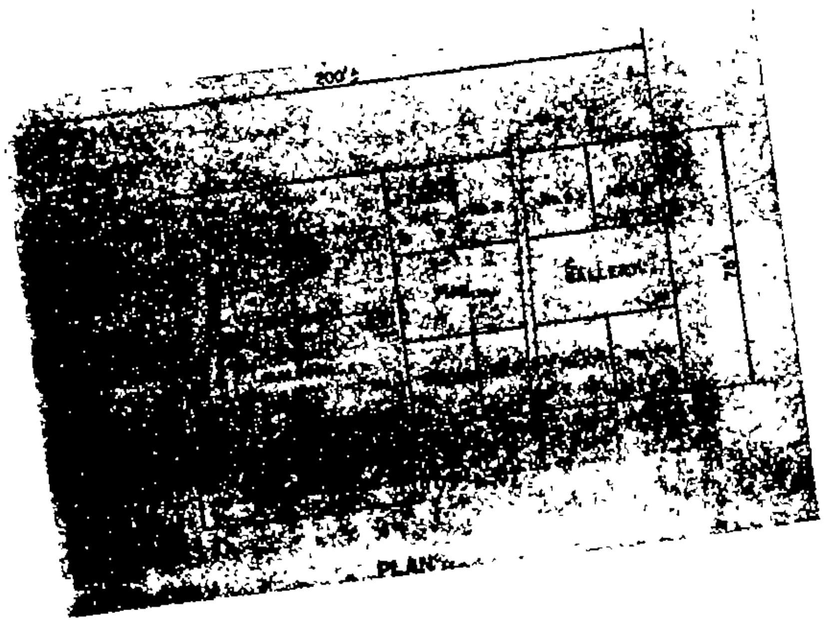


Figure 8
Plan - High Rate Filtration Installation
(100 MGD)

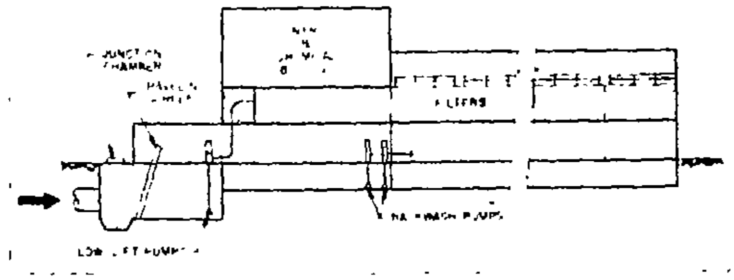
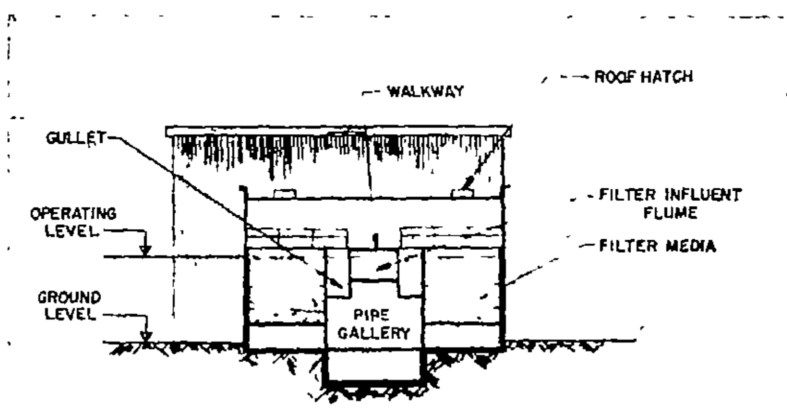


Figure 9
Longitudinal Section High Rate
Filtration Installation (100 MGD)



CROSS SECTION

Figure 10
Cross Section - High Rate
Filtration Installation (100 MGD)

Before entering the pumping station, the combined sewer overflow would pass through a bar rack (screen) for removal of coarse materials which might cause problems in the operation, maintenance or wear of the low lift pumps. In certain locations, where consistent with local topography and sewer invert, a low lift pumping facility may not be required.

The combined sewer overflow from the low lift pump station would enter a treatment building and be delivered to drum type screening units. The wastewater would be introduced into the center of the drum type screen and would pass through the screening mesh into the influent channel to the filters. A gravity type design, i.e., open filtration units, is proposed. The water would be introduced at the top of the filter and flow downward through the filter bed. The plant effluent could be discharged by gravity to the respective receiving water body.

Filtered wastewater would serve as a source of water for backwashing filters after the overflow has attenuated to a sufficient degree. The filtration building would be provided with low pressure air blowers as a source of backwash air. Backwash pumps would be located in the filtration facilities to deliver water to the filters for backwashing. The treatment building would also include a control area, office space, a polyelectrolyte feeding set-up, and a system for adding hypochlorite to filter backwash water for the prevention of slime growth on the filter media. The operation of the high rate filtration facility would be completely automated, and could be left unattended, except for routine maintenance and periodic delivery of chemicals. In full size treatment systems, chlorine feed for disinfection could be incorporated into the filtration facilities.

Dirty backwash effluent from the filtration facilities and screenings would be directed into the interceptor running to the sanitary sewage treatment facility. The concentrated solids from the drum screening units would be passed first through a grinder, and then through a trash basket or classification device to insure that very coarse, settleable material is not returned to the sewer system. Sludge handling facilities should not be located at the filtration site, as this would prove very costly. Centralization of material handling facilities has always proved most economical; as an example, the Southerly Wastewater Treatment receives sludge from another plant in Cleveland.

For filter backwashing, two types of process control should be considered: the first parameter would be total head loss through filter bed, and the second would be effluent suspended solids concentration.

For measuring the filter head loss, each filter would be equipped with a differential pressure transmitter to continuously sense the loss of head across the filter and transmit a pneumatic signal linearly proportionate to this head loss to a central control panel. When the filter head loss would reach a preset value, the differential pressure switch associated with the filter would be actuated. A contact in this switch would open a stepping switch circuit and the filter would start to backwash.

An alternate, filter backwash control could be achieved with an effluent suspended solids monitor. A continuous reading, light scatter type suspended solids meter would be installed in each filter effluent pipe to continuously measure the suspended solids concentration and transmit the reading to a recorder at a central control panel. When the filter breakthrough would suddenly take place and the suspended solids concentration indicator would reach a preset level, then a micro switch would be activated and an alarm would be initiated. The operator would check the filter performance condition and start to backwash the filter.

Principal advantages of the proposed system are: high treatment efficiencies, automated operation, and limited space requirements as compared with alternate flotation or sedimentation systems.

COST DATA

Estimated total construction costs (ENR=1470) of a filtration plant for treating combined sewer overflows range from $830,000 for the 25 MGD capacity to $3,754,000 for 200 MGD capacity at design rate of 24 gpm/sq ft.

Estimated annual cost data ranges from $97,270 per year for a 25 MGD capacity plant to $388,210 per year for a 200 MGD capacity plant. Annual treatment costs utilizing the high rate filtration process are due primarily to interest and amoritzation charges, and are less affected vby the volume of combined sewer overflow to be treated annually.

These costs do not include disposal of waste screenings and filter backwash since the proposed system would discharge these to the municipal sewage treatment plant. Assuming an average of 200 mg/l of solids removed and a combined sewer overflow treatment plant operation of 300 hours per year, solids processing and disposal costs incurred by the municipal sewage treatment plant could range from 3 to 35 percent of the total annual charges for the combined sewer overflow treatment facility.

DUAL PURPOSE OF UTILIZATION OF HIGH RATE FILTRATION PROCESS

The selected media for combined sewer overflow treatment was also evaluated in terms of its capacity for polishing secondary effluent under another research contract. Test data has confirmed the applicability of this combined sewer overflow media to reducing suspended solids, BOD, and phosphate to low residuals.

In Cleveland, the total duration of the overflows from the combined sewer system is approximately 300 hours per annum. This indicates the possibility of utilizing dual purpose treatment plants based on the high rate filtration process. Such installations would treat combined sewer overflows when they occur, and in between such periods, for over 95 percent of the time, the filtration process would treat other wastewaters depending on the location of the process.

For a high rate filtration process for combined sewer overflow treatment located in the area of the domestic wastewater treatment plant, the filtration process can be utilized for polishing the treatment plant effluent as well as to protect the effluent quality during plant overloading or process malfunction.

The economical benefits of such dual purpose utilization of the high rate filtration process should not be overlooked.

SECTION VII

SCREENING/DISSOLVED - AIR FLOTATION

TREATMENT OF COMBINED SEWER OVERFLOWS

by

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Introduction

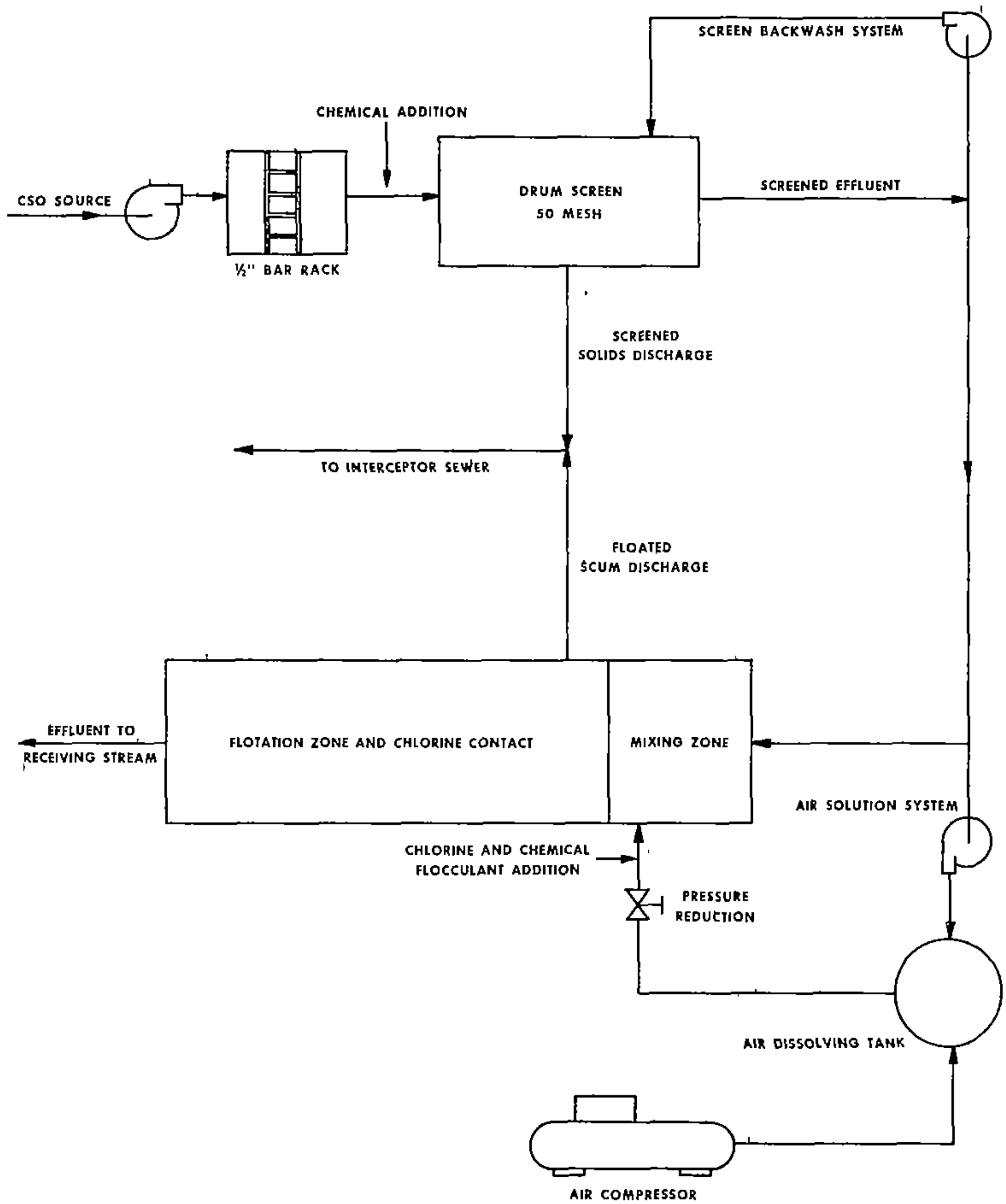
The problem of combined sewer overflows (CSO) has been recognized as a significant pollution problem in recent years (1). Large amounts of untreated pollutants find their way into our water courses through this route. The abatement methods dealing with this problem are sewer separation, storage, treatment, or a combination of these. The cost of separating the sewers is prohibitive and this method is not considered as an economical solution to the problem. A great deal of literature has been published since 1964 which describes the characteristics of (CSO) (2). Based on the data published, it has now been established that a major portion of the pollutional substances in CSO is particulate in nature. This indicates that an efficient solid/liquid separation process can be expected to provide an effective treatment of CSO. It was the mission of the Environmental Sciences Division of Rexnord Inc. to develop an effective and economical solid/liquid separation process under a program sponsored by the U.S.Environmental Protection Agency.

A combined sewer outfall near Hawley Road in the west-central portion of Milwaukee, Wisconsin was selected as a source of combined sewer overflow for the bench scale studies. This outfall services a 495 acre residential area. It was determined that approximately 42% of the area was impervious, i.e. streets and parking areas, house roofs etc. The calculated value of the runoff coefficient was 0.40 and it compares well with the values reported in the literature (3). The drainage area comprises of mostly one and two family dwellings with an estimated density of 35 people per acre. No manufacturing industries are located within the drainage area except some small business shops. Bench scale tests were conducted on 14 separate overflow samples to define the quality of the Hawley Road outfall and to evaluate the various potential treatment processes. The evaluatory tests included screening with various sized media, chemical oxidation, flotation and disinfection. It was determined from these tests that chemical oxidation of the raw CSO did not appear technically and economically feasible (4). However, the results of the screening and dissolved-air flotation tests were encouraging. These tests served as the design basis of a 5 MGD test facility at the Hawley Road outfall utilizing screening and dissolved air flotation.

Design of the Treatment System

The process schematic of the proposed treatment system is shown in Figure 1. The raw overflow is pumped from the sewer to a half inch manually cleaned bar rack. The purpose of the bar rack is to remove large objects which may clog or damage the finer screen downstream. The flow then enters a 50 mesh (approximately 300 micron) drum screen. The basic screen is fabricated from mild carbon steel while the screening media is a 304 stainless steel. The screen is an octagonal shaped drum with an effective diameter of 7.5 ft. and 6 ft. length. The total screen area is 144 sq. ft. with wetted screen area ranging between 72 and 90 sq. ft. depending upon the head loss across the screen. The design hydraulic loading for the screen is 50 gpm/sq. ft. and a maximum head loss capacity of 14 inches. The drum speed can be varied in the range of 0.5 to 5.0 rpm.

Screened water is used to backwash the screen. The solids which are removed from the screen are collected in a hopper and are then routed to the sanitary sewer. The screened effluent is split into two portions. A major portion of the flow goes directly to the flotation tank while the remainder of the flow



SCREENING/FLOTATION FLOW DIAGRAM

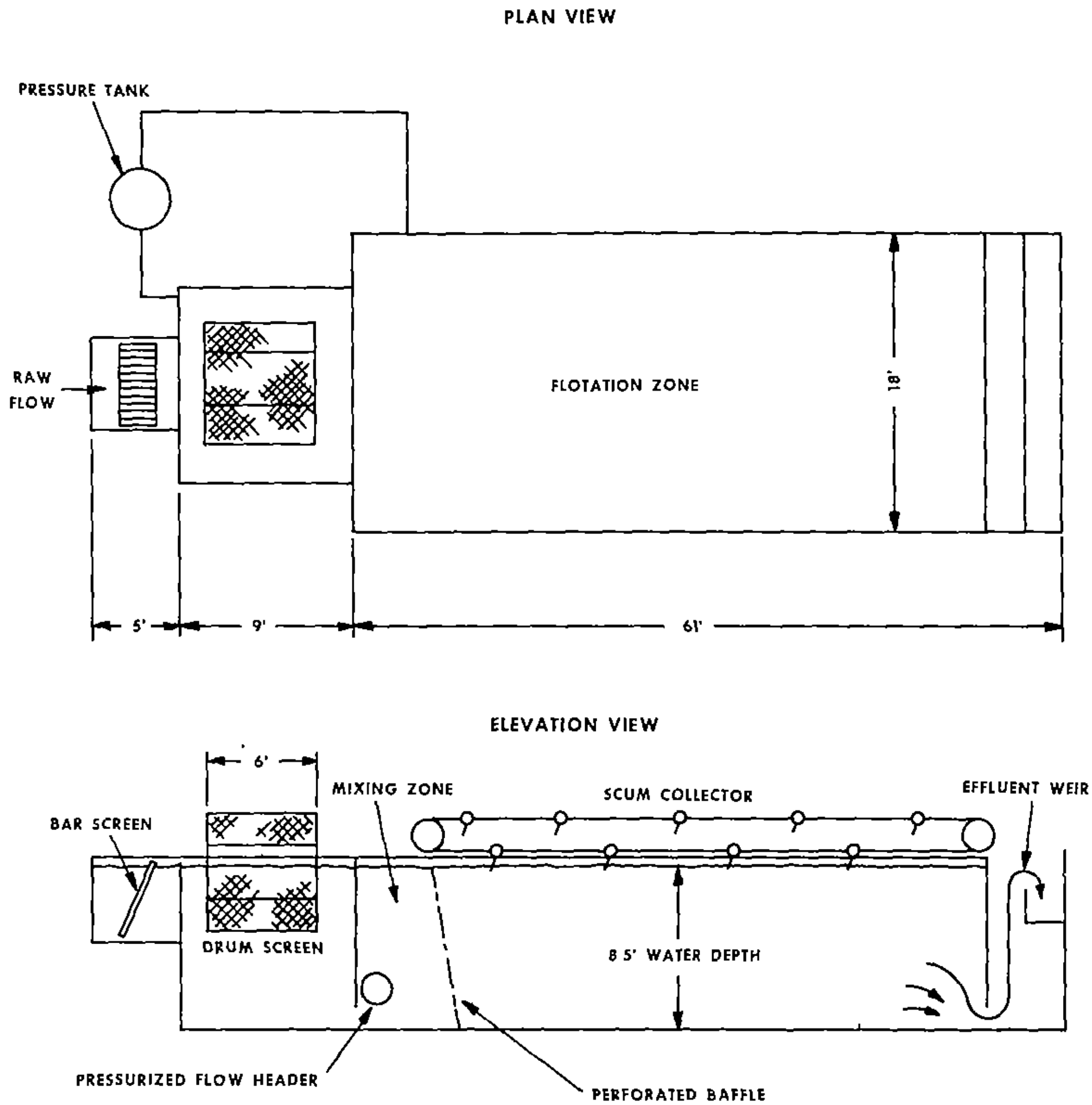
FIGURE 1

(approx. 20%) goes to a pressure tank where it is mixed with air under pressure (approx. 50 psi). The pressurized air-water stream is then brought into contact with the bulk of the raw flow at atmospheric pressure in a mixing zone. The dissolved air comes out of solution in the form of tiny bubbles (50-100 micron size) in the mixing zone and these bubbles attach themselves to the suspended matter in the waste water. The mixed flow then passes through a distribution baffle and into the flotation tank where solid/liquid separation occurs. The scum which floats to the top is then scraped into a trough via skimmers and is routed to the sanitary sewer. The treated effluent is discharged to the Menomonee River.

The main details of the treatment system are shown in Figure 2. Flexibility was provided in the design so that the flotation zone could be segmented for evaluating various hydraulic overflow rates. Chemical flocculants when utilized were added to the raw waste as it entered the drum screen or in the pressurized flow stream after the pressure reduction valve. Chlorine was also added in the pressurized flow stream for disinfection of the CSO. The entire system was automated and was put into operation by sensing a-pre-set level of the waste water in the sewer.

Operation of the Demonstration System

The system was operated on 55 separate combined sewer overflows during 1969 and 1970. The quality characteristics of these overflows are seen in Table 1. About 20 percent of the overflows exhibited the first flush phenomenon, which was either caused by high rainfall intensity or a length of time greater than four days between overflows. After the first flush diminished, the quality of the overflow was remarkably constant for each storm. The 95% confidence ranges for the extended overflows were only about 10-



DEMONSTRATION SYSTEM DETAILS

FIGURE 2

TABLE 1

COMBINED SEWER OVERFLOW CHARACTERISTICS AT HAWLEY ROAD[1]

Analysis	First Flushes[2]		Extended Overflow[3]
Total Solids (mg/l)	861 ± 117		378 ± 46
Total Volatile Solids (mg/l)	489 ± 83		185 ± 23
SS (mg/l)	522 ± 150		166 ± 26
VSS (mg/l)	308 ± 8.3		90 ± 14
COD (mg/l)	581 ± 92		161 ± 19
BOD (mg/l)	186 ± 40		44 ± 10
Total Kjeldahl Nitrogen (mg/l)	17.6 ± 3.1		5.5 ± 0.8
pH	7.0 ± 0.1		7.2 ± 0.1
Total Coliform (individuals/ml)	$142 \times 10^3 \pm 108$		$62.5 \times 10^3 \pm 27$
Dissolved COD/Total COD[4]	--	0.34 ± 0.04	--

[1] Ranges shown at 95 percent confidence level.
[2] Represents 12 overflows.
[3] Represents 44 overflows.
[4] Represents 34 overflows.

15% of the mean value as compared with 20-25% for the first flush data. The dissolved organic fraction (measured as chemical oxygen demand) was approximately one third of the total organic load in the raw combined sewer overflow. This showed that a large portion (2/3 of the total) of the organic pollutants was of a particulate nature which would be amenable to treatment via screening/ dissolved-air flotation.

The variables evaluated during operation included hydraulic loading and drum speed for the screening operation, and surface overflow rate, pressurized flow rate, operating pressure, and flocculant dosages for the flotation system. The optimum operating conditions based on the treatment of 55 CSO are given in Table 2. The optimum solids loading rate at a drum speed of 4.7 rpm and a head loss of 12" was 1.2 pounds of dry solids removed per 100 sq. ft. of screen area. This loading could possibly be increased by increasing the allowable head loss differential. The hydraulic throughput rate was in the range of 40-45 gpm/sq.ft. This rate again can probably be increased depending upon solids loading. It was found that no statistical difference could be shown in the removal efficiencies by increasing the pressurized flow rate up to 45 percent of the raw flow, or by increasing the operating pressure to 60 psi. A pressurized flow rate of 20% of the raw flow at 50 psi was recommended for future designs. The air usage was approximately one cfm per 100 gpm of pressurized flow. The overflow rate at which removal efficiencies were satisfactory and the capital cost still reasonable was 3.3 gpm/sq.ft. Floated scum concentrations generally ranged between 0.7 and 1.4% of the raw flow. The chemical flocculants utilized during this study were $FeCl_3$ and a cationic polymer (C-31, Dow Chemical Co.). The selection of these chemicals was based on the results of a series of bench scale jar tests. The optimum chemical dosages were found to be 20 mg/l $FeCl_3$ and 4 mg/l of C-31.

TABLE 2

OPTIMUM OPERATIONAL CONDITIONS

Characteristics	Operational Condition
Screening	
Backwash	0.7 - 1.0% raw flow
Head Loss	12 in. water
Rotation Speed	4.7 rpm
Submergence	50 - 63%
Hydraulic Throughput Rate	40 - 45 gpm/sq. ft.
Flotation	
Floated Scum	0.75 - 1.41% raw flow
Pressurized Flow	20% raw flow
Operation Pressure	50 psi
Overflow Rate	3.3 gpm/sq. ft.
Chemical Dosage	20 mg/l $FeCl_3$
	4 mg/l cationic polyelectrolyte

The performance of the 50 mesh screen alone is summarized in Table 3. The pollutant removals (measured in terms of suspended solids, volatile suspended solids, COD and BOD) ranged between 33-39% for the first flushes and between 26-34% for the extended overflows. The slightly higher removal efficiencies for the first flush overflows is probably a result of the screening-filtration phenomenon that occurs during these high pollutant loading periods.

The total removal efficiencies for the combined screening/flotation system are shown in Table 4. The pollutant removals ranged between 35-48% without flocculating chemicals. However, the removal efficiencies were significantly enhanced on the addition of flocculating chemicals and ranged between 57-71%. Removals during the first flushes were similar to the results for extended overflows with chemical addition. The average effluent quality experienced with chemical addition and that can be expected via screening/flotation treatment is shown in Table 5. These values compare favorably with many secondary sewage treatment effluents.

Future Design Considerations

The data presented so far had been based on the results of two operational seasons, 1969 and 1970. Research was continued on this treatment facility during 1971 to obtain additional design data for the optimization of the screening and dissolved-air flotation processes in order to improve upon the effluent water quality of the treated combined sewer overflows.

Laboratory bench scale tests have indicated that changing the split flow mode of dissolved-air flotation to effluent recycle mode of operation may enhance the effluent water quality significantly. This change may require the operation of the flotation

TABLE 3

PERCENT POLLUTANT REMOVALS BY SCREENING*

Characteristics	First Flushes	Extended Overflow
SS	36 ± 16	27 ± 5
VSS	37 ± 18	34 ± 5
COD	39 ± 15	26 ± 5
BOD	33 ± 17	27 ± 5

* Values given at the 95 percent confidence level.

TABLE 4

PERCENT POLLUTANTS REMOVALS BY SCREENING/FLOTATION TREATMENT*

		Extended Overflows	
Characteristic	First Flushes	Without Chemicals	With Chemicals
SS	72 ± 6	43 ± 7	71 ± 9
VSS	75 ± 6	48 ± 11	71 ± 9
COD	64 ± 6	41 ± 8	57 ± 11
BOD	55 ± 8	35 ± 8	60 ± 11
Nitrogen (total Kjeldahl)	46 ± 7	29 ± 8	24 ± 9

* Values shown in a 95 percent confidence range.

TABLE 5

EXPECTED AVERAGE EFFLUENT QUALITY AT HAWLEY ROAD

Analysis	Value (mg/l)
SS	48
VSS	26
COD	69
BOD	20
Nitrogen (total Kjeldahl)	4.2

system at reduced overflow rates and could therefore increase the flotation area requirements by approximately 20%.

Also, several other chemical flocculant combinations have shown promise over the ferric chloride - C-31 polymer combination utilized during the 1969 and 1970 operational seasons. Use of powder activated carbon along with screening/dissolved-air flotation has also shown some merit. The economics of these concepts for an optimum cost benefit relationship still need evaluation. These evaluations are a part of the proposed modifications to the Hawley Road treatment facility. It is anticipated that these considerations will be evaluated on the modified Hawley Road treatment facility during the 1973 operational season.

Racine Root River Project

Encouraged by the promising results of the Hawley Road demonstration facility, a search was made to find a site where the feasibility of utilizing screening/dissolved-air flotation could be demonstrated on a full scale for the treatment of combined sewer overflows. The City of Racine, Wisconsin was indicated to be an ideal site for such a project. Racine is a city of approximately 100,000 people located on Lake Michigan, approximately 30 miles south of Milwaukee. The Root River, a stream having a mean annual discharge of approximately 100 cfs flows through the city and serves as a receiving body for runoff from much of the northern half of the city. There are approximately 700 acres of land having combined sewer systems in this area. In the 3.7 miles of Root River through the city, there are 36 combined sewer overflow points and 17 storm water discharges to the river. It was estimated that the cost of separation of the existing combined sewer areas in Racine would be 10-13 million dollars. The estimated cost of installing the screening/dissolved-air flotation treatment plants at the various outfalls was 4 million dollars. Thus significant savings were evident in going for the screening/dissolved-air flotation route for the treatment of combined sewer overflow problem in the City of Racine.

In April of 1970 a grant application was submitted to the U.S. Environmental Protection Agency. Under the terms of this proposal the funds would be rendered by the federal government, State of Wisconsin, and the City of Racine. The technical approach proposed for meeting the project objectives includes the following elements:

1. Quantitative measurement of the effects of treating storm-water discharges and combined sewer overflows to a selected stretch of the Root River as a function of river water quality.
2. Detailed cost/performance analysis for treatment in the selected stretch of the Root River.
3. Full-scale verification of the combined sewer/water quality mathematical model developed under EPA Grant No. S800744.
4. Application of results from Items 1, 2, and 3 above to the following determinations:
   a. Process adequacy of treatment system as an alternate to combined sewer separation in 700 acre area of central Racine, Wisconsin.
   b. Cost/benefit relations of treatment system for the subject 700 acre area in central Racine, Wisconsin.
   c. Validity of the EPA Storm Water Management Model for application to problems of any given area.
   d. Process, design, operation and application criteria for treatment method as alternate to combined sewer separation in any given area.

This proposal was approved by the City of Racine and in July, 1970 the project was approved and funded by EPA. The City of Racine became the grantee and a subcontract was awarded to the Environmental Sciences Division of Rexnord Inc. to conduct the study program. Also the consulting engineering firm of Donohue and Associates of Sheboygan, Wisconsin was retained by the City of Racine for the engineering design of this project.

An engineering study was conducted by the Rexnord Environmental Sciences Division and Donohue and Associates in cooperation with the Racine City Engineer's Office for the purpose of choosing a site which would allow for maximum CSO treatment within the project dollars. The two selected locations which contained the overflow outfalls from a large percentage of the total area and where these discharges were in close proximity to one another were:

1. Site I - South of Dodge Street between Chatham and Michigan Streets
2. Site II - South of Dodge Street between Main and Wisconsin Streets

The overflows in these locations drain from a total area of 450 acres of combined sewers. Site I was designed for a treatment capacity of 14.1 mgd and Site II for 44.4 mgd.

System Design

Two full scale SDAF systems have been installed in Racine for treatment of combined sewer overflow. The design criteria for each of the various elements is shown in Table 6. The systems have been designed for completely automatic startup, operation and shutdown.

The two systems are similar in function and differ only in design capacity. A schematic diagram of the larger system is

TABLE 6

DESIGN CRITERIA - SCREENING/AIR FLOTATION TREATMENT SYSTEM
RACINE, WISCONSIN

Item	Site #1	Site #2
Contributing area (acres)	82.5	364.2
Design Storm Intensity (inch/hour)	0.5	0.5
In-Sewer Storage (gallons)	--	600,000
Design Flow for Treatment System (MGD)	14.13	44.4
Bar Screens		
Mechanically cleaned and located Just Upstream of Pump Sump	Yes	Yes
Drum Screens		
Parallel Operation, automatic bypass to flotation tanks should all screens clog		
Number of screens	2	4
Length (feet)	7	10
Diameter (feet)	8	8
Filter Media Stainless Steel - 50 mesh, .009 inch wire		
Screen Backwash flow gpm (when operating)	210	675

TABLE 6 CONTINUED

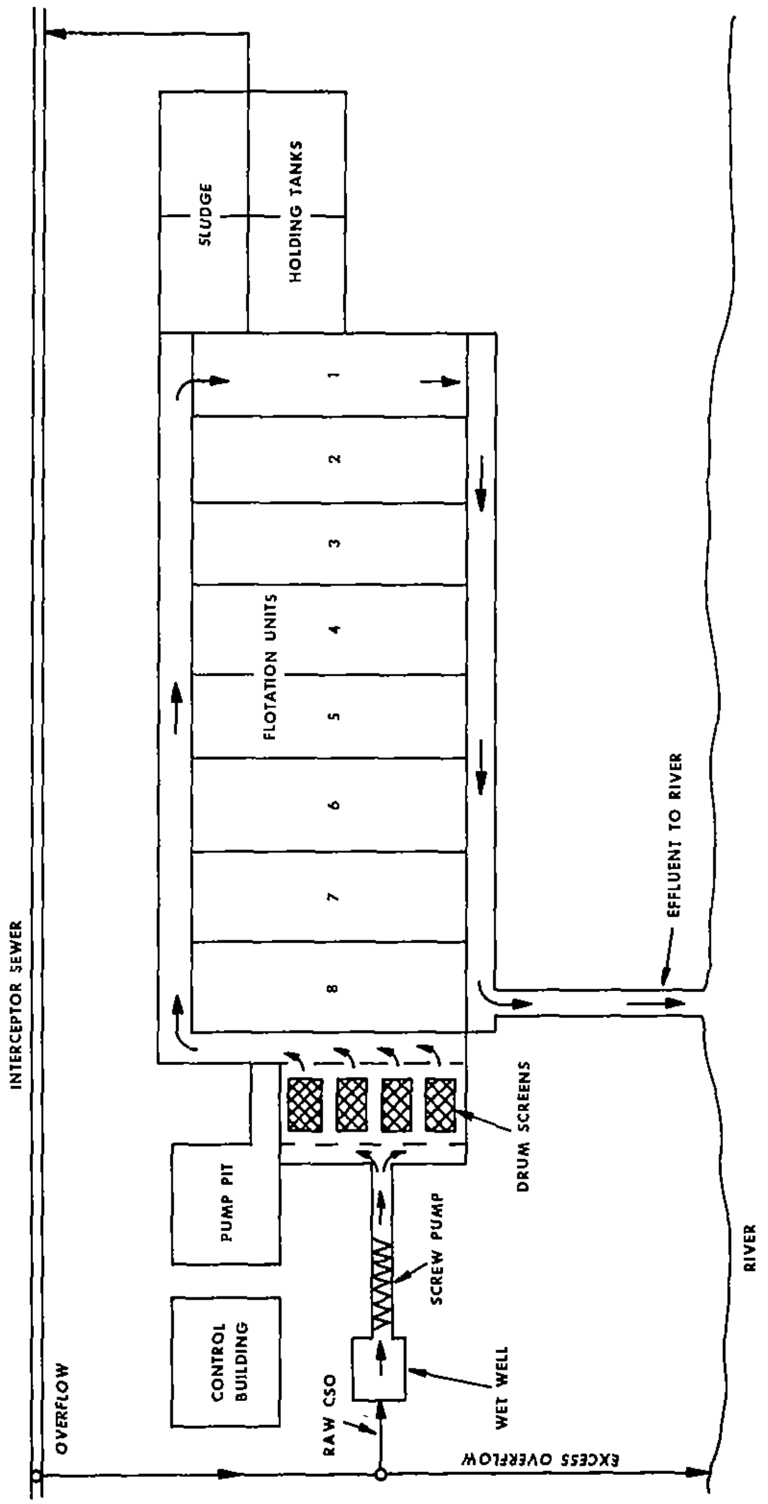
Item	Site #1	Site #2
Flotation System		
Operation - Each tank reaches 70% maximum flow before the next tank is put into use.		
Number of tanks	3	8
Surface overflow rate - gpm/ft^2	3.5	3.5
Pressurized flow - gpm/tank	650	770
Scum Removal - timer controlled		
Surface skimmer to scum trough -		
Screw conveyed to sludge holding tank		
Chemicals		
Chlorine - maximum concentration mg/l	20	20
$FeCl_3$ - maximum concentration mg/l	25	25
Polyelectrolyte - concentration		
Dependent on specific polyelectrolyte		
Sludge Storage		
1.5% of design flow for 3 hour duration		
Volume - cubic feet	3,500	11,030
Disposal to sanitary sewer by gravity		
Drain following storm		

shown in Figure 3. Upon sensing a high level in the overflow sewer, the system is placed into operation. Raw overflow enters the plant through a mechanically cleaned bar screen located in the wet well. A by-pass weir is provided for storm flows in excess of the design capacity. Flow entering the wet well is pumped by means of a spiral screw pump through a Parshall flume and into the screening chamber. The output of the flow recorder/ totalizers are used to provide a proportional signal for pacing the chemical feed equipment. Ferric chloride is added to the wastewater upstream of the screens. Chlorine and polyelectrolyte are added downstream of the screens.

Each of the drum screens is equipped with 50 mesh stainless steel screens. The screens are backwashed at a preset headloss level. Solids removed on the screen are conveyed to a sludge holding tank by means of a screw conveyor which runs along the head end of the flotation tanks.

Effluent from the drum screens is diverted to the flotation tanks by means of a series of weirs and orifices. The inlet system is designed so that the tanks are filled in series. This enables the utilization of only as much tankage as is actually required by the storm flow. Screened effluent is used as the source of pressurized flow.

Scum produced in the air flotation tanks is skimmed to the head end of the tanks where it is conveyed to the sludge holding tanks by means of a screw conveyor. All sludge generated during a storm is held in the holding tanks until after the storm subsides and then is discharged to the interceptor sewer. At some future date it may prove fruitful to provide onsite dewatering facilities rather than return the concentrated sludge to the sewer system.



SCHEMATIC LAYOUT OF THE TREATMENT SYSTEM FOR SITE NO. 2

FIGURE 3

The flotation tank effluent which has been chlorinated will be discharged directly to the Root River.

Following a storm all of the sludge, as well as the contents of the flotation tanks will be discharged to the adjacent sanitary interceptor sewer. The system will then be ready for the next storm.

Special Considerations

Certain special considerations have been made in order to insure optimum use of the system. A floodgate was installed in one of the overflow sewers to provide approximately 600,000 gallons of in-system storage. This storage capacity will be utilized when the treatment facility reaches full capacity.

In addition, the system has been equipped to be completely self-draining. This will enable use of the system during periods of snow melt and cold weather. A roof has also been provided to prevent floc breakup during heavy rains.

Costs

The cost for the Racine SDAF system is $30,000 per mgd installed capacity. A detailed cost breakdown is given in Table 7.

Racine Program

A two year system evaluation and optimization is scheduled to begin on April 1, 1973. The intent of this program is to fully evaluate the installed facility, validate the EPA Stormwater Management Model and determine the effect of the system on water quality in the Root River.

TABLE 7

COST OF SCREENING/DISSOLVED AIR FLOTATION

Capital Costs

Cost per MGD Capacity	\$30,000
Cost per Acre*	\$ 3,900

* Based on 0.5"/hour runoff rate

Operating Costs

		¢/1000 gallons
Power		0.54
Chemicals		2.51
Maintenance		0.04
	TOTAL	3.09¢/1000 gallons

Based on plant capacity of more than 30 MGD
and 40 hours per month operation.

Acknowledgement

The work in this paper for the Hawley Road Demonstration Facility was sponsored by the U.S. Environmental Protection Agency. The implementation of the findings of the Hawley Road project as applied to the Racine Root River Project were undertaken through the joint sponsorship of the U.S.E.P.A., State of Wisconsin and the City of Racine. Portions of this paper have been derived from two publications: 1) Screening/Flotation Treatment of Combined Sewer Overflows, EPA Project Report by Ecology Division, Rex Chainbelt Inc., WPCR Series 11020 FDC, January, 1972, and 2) Treatment of Combined Sewer Overflows by D.G.Mason, JWPCF, December, 1972.

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SECTION VIII

HIGH-RATE DISINFECTION OF COMBINED SEWER OVERFLOW

by

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The bacteria content of combined sewer overflow has been reported to be as high as 30 million total coliform/100 ml and 3 million fecal coliform. These levels are 1,000 to 10,000 times the allowable concentration in secondary effluents and similar restrictions have been considered for combined sewer overflows. The techniques used to remove suspended solids have in themselves no ability to remove or kill coliform. Thus bacteria kills of 3 to 4 logs (that is, 99.9% to 99.99%) are required as a separate operation for combined and separate sewer overflows.

As reported by others (1) (previous speakers) it may be possible to achieve a suitable bacteria kill with high chlorine dosages within certain types of solids removal devices so that no separate contact chamber will be required. Considerable more work needs to be done over a broad range of flow rates before the proposed advantage of dual use of this volume can be utilized on full scale plants. *It is anticipated that required bacteria kills may not be obtained at* low flow rates.

The special design considerations required to cope with the very high instantaneous overflow rates previously mentioned (this morning) for removal of suspended solids and organic matter hold for the disinfection equipment as well.

Conventional chlorine contact chambers installed at sewage plants are sized to provide 15 to 30 minutes detention which would require considerable area (about 1 acre per 250 acres drained at 1.0 cfs/acre). Operating close to their design rate as determined by the 2 to 1 diurnal flow variation, these basins, as often as not, fail to achieve the required bacteria kills. During the initial filling, these sewage contact chambers do not, and are not expected to, perform. A contact chamber sized to provide 15 minutes residence for a peak stormwater overflow rate would never be filled to its operating level during most storms. The operation of conventional 15-30 minute contact chambers in combined sewer overflow would be uncertain at best.

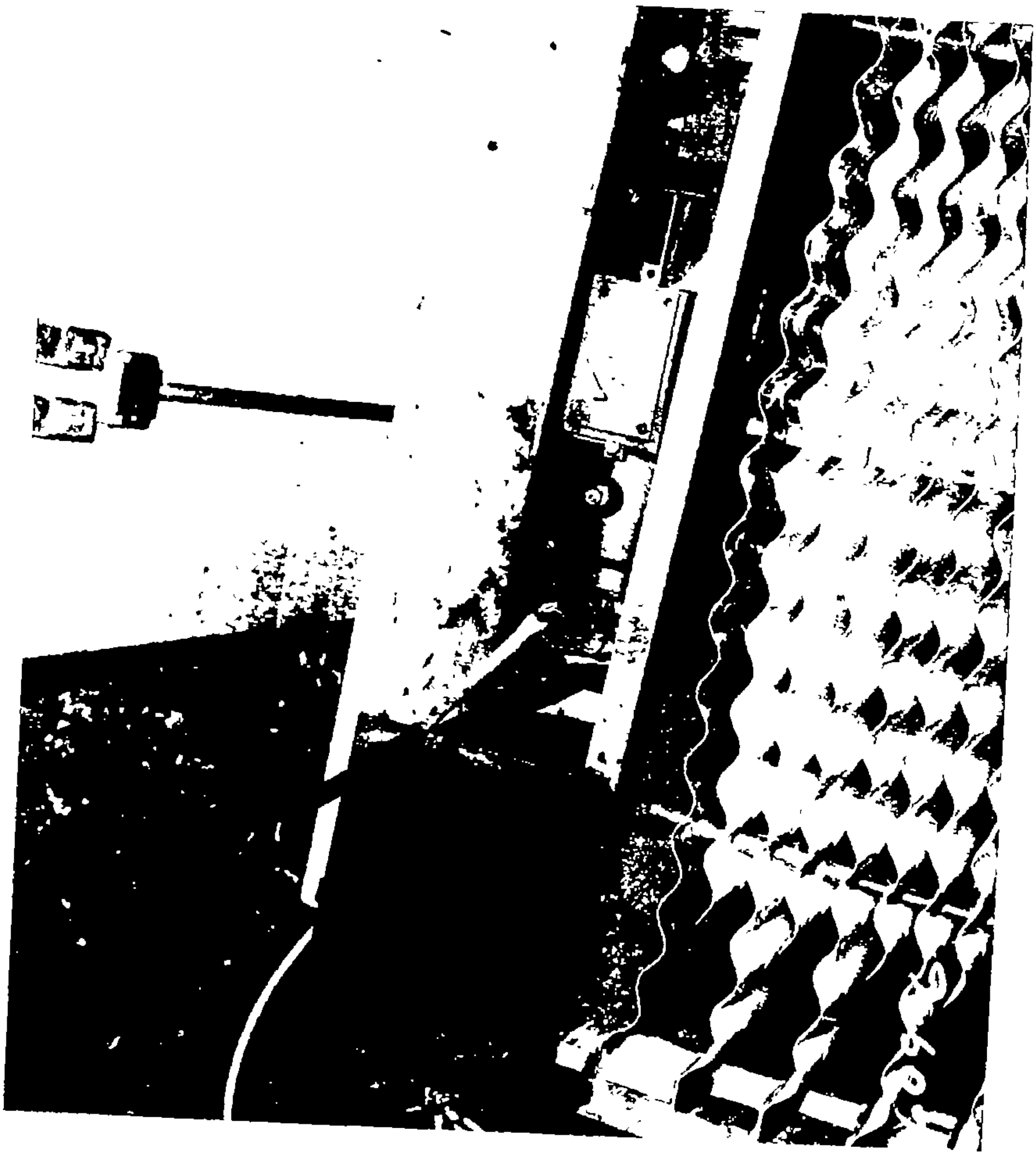
Our work on disinfection as well as the work of others (2) (3) was performed in pilot size contact chambers at a constant flow rate. That is, these chambers have not been tested at the wide (20 to 1) variations in overflow rate anticipated for a full scale chamber in stormwater service. As will be seen later, the assumption that performance of a contact chamber will be as good, if not better, at lower flow rates as it is at the higher rates is questionable even though the contact time is longer.

We have made five disinfections of combined sewer overflow while the storm was in progress. We achieved 99.99% kill (4 logs) with chlorine dosages (10 ppm) in 120 seconds. The flow rate through our units - we have two identical units - was 20 gpm. In every case, both total and fecal coliform were reduced to below 10 cells/100 ml. This performance was obtained on both the raw overflow before Microstraining and the microstrained effluent. The 3 minute chlorine demand was surprisingly uniform at about 3 ppm for the microstrained effluent and somewhat higher for the raw stormwater.

One of these chambers is shown in Figure 1. They were designed to

Figure 1

Intensely Mixed Chlorine Contact Chamber



ensure that the hypochlorite was promptly and well mixed with the stormwater. More important, or equally important, they were designed to ensure a high degree of small eddy turbulence in the passages of the contact chamber.

We attribute the extraordinarily high kill rate of these chambers to the turbulence during contact time.

The very recent literature (Collins et al (2), Kruse et al (3) and the Dow work (4)) reports several instances of laboratory studies on sewage and stormwater disinfection where similar extraordinary kill rates have been observed. Examination of the apparatus and the procedure used in these studies reveals that very high turbulence existed during these studies as well.

In one case - a beaker study by Kruse et al (3), a high stirring rate was used to demonstrate the advantage of prompt and thorough dispersion of the chlorine. Very high (4 - 5 logs) kill rate of bacteria was observed in 2 minutes when the fast stirring rate (i.e., "fast mix") condition was sustained throughout the whole study. Much poorer performance (only 1 - 2 logs in 2 minutes) was obtained at the same dosage when the more normal mixing regime of a few seconds fast mix followed by 15 minutes slow mix was used. It is of great importance that, in this study, virus were killed at high rate under the sustained fast mix condition for a few minutes whereas there was minimal virus kill even with prolonged slow mixing.

In the case of the Dow EPA (4) study, a long 1,500 ft tube was selected as a flow thru contact chamber. This configuration was apparently selected to permit precise collection of samples after a specified contact time and to avoid the problem of "true contact time vs nominal contact time" encountered in conventional flow-thru chambers due to short circuiting and back mixing, etc. The tube diameter was fortunately small enough (1-1/2" ID) relative to the flow (8 gpm) to cause turbulent flow. High kill rates were achieved with nominal chlorine dosages. For example, a 4 log kill at 2 minutes with 8 ppm dose. It is pertinent to the subject of the present paper that the Dow investigators assumed that the performance achieved in their turbulent laboratory chamber could be duplicated in a much less turbulent full scale chamber. This is, as we will see, a very questionable assumption although it is routinely made in applying laboratory data to design of full scale chambers.

Collins et al (2) also used a small diameter tube reactor in which very high velocities (13 ft/sec) caused very high turbulence. Here also, high rate kills on sewage of over 3 log in 1 minute at dosage 5 ppm were obtained. These investigators recognized, and qualitatively demonstrated (or probably the reverse order), the effect of turbulence during contact time by comparing the performance of their highly turbulent tube reactor with a gently stirred batch reactor which achieved less than 1 log kill in 1 minute at same dosage.

The basic design scheme for the pilot chambers used in our study was developed in early 1968 to support a cost estimate for 2 minute contact chambers. This cost estimate was included in the report entitled "Microstraining and Disinfection of Combined Sewer Overflows Phase I" (5).

The need for 1-2 minute disinfection equipment to complement the high rate solids removal equipment in combined sewer overflow service was apparent.

A survey of the literature to 1968 revealed no indication that 2 minute disinfection was reasonably possible.

The literature survey did reveal the situation that full scale chambers with 15-30 minutes volume routinely failed to approach the performance predicted by batch type jar tests in the laboratory. In particular, the disinfection performance of full scale chambers in cold weather was reported to be poorer than expected by temperature difference. That performance did not improve at less than design flow rate; i.e., longer contact time. These, and other considerations, suggested the parallels between disinfection rate and the flocculation rate in water treatment where a similar situation existed.

Fortunately there is a method slowly gaining acceptance by which the effect of mixing intensity on flocculation performance in laboratory and in the field can be correlated. The use of the velocity gradient (G) parameter as a measure of mixing intensity was first proposed by Camp & Stein (6) in 1943 to explain floccuation rates. They showed that velocity gradient is the difference in velocity of two parallel flowing planes of fluid in ft/sec divided by the distance between the planes in feet. Further, they showed that it was a measure of the opportunities for particle to particle (molecule) collisions per unit time per unit volume.

The product of velocity gradient times actual contact time (GT) is the number of opportunities for collision per unit volume during the flocculation operation. It follows that the GT product is proportional to the fraction of total number of particles (molecules) initially present which are actually engaged in a collision during the operation.

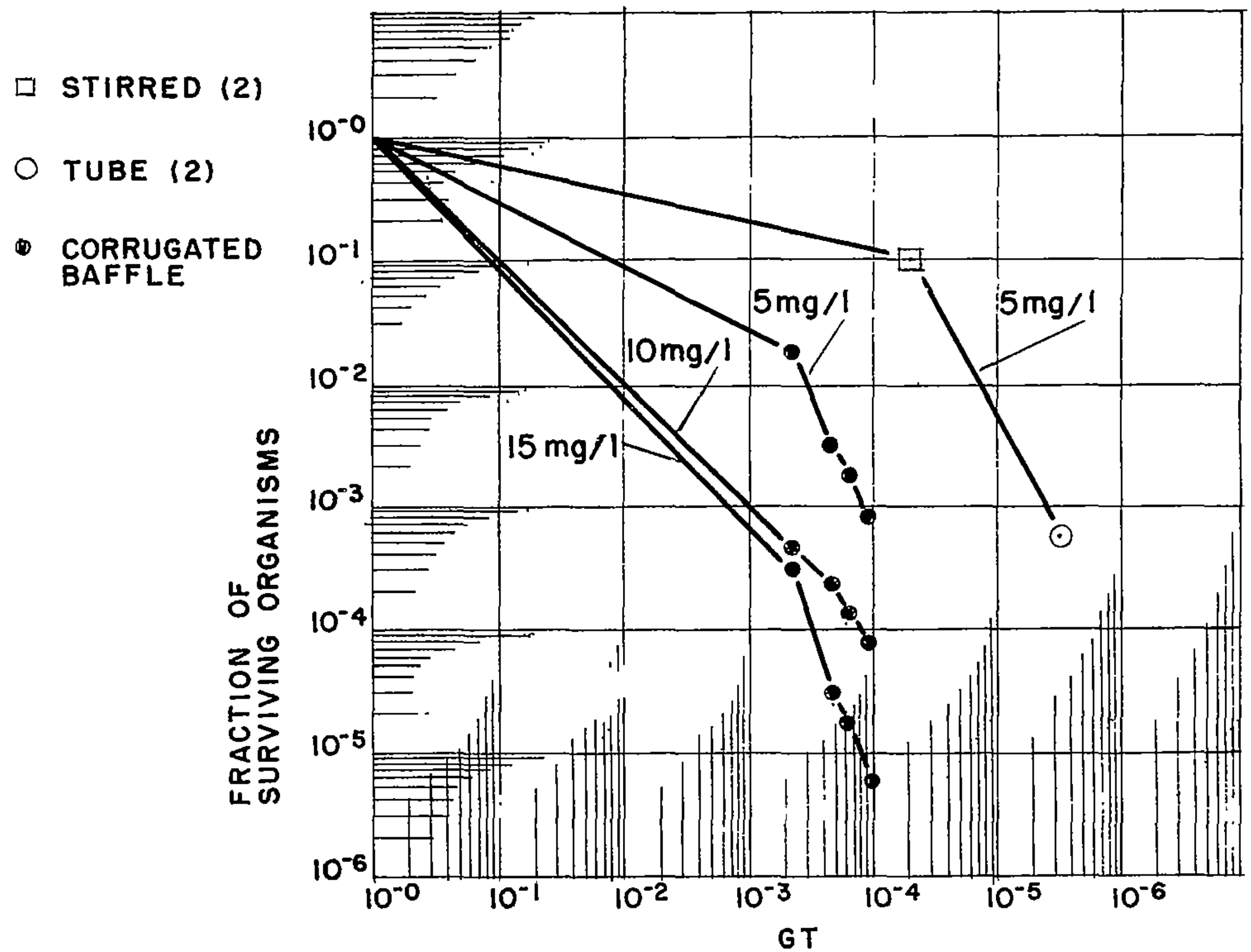
Several studies (7) (8) have shown that the reduction of the number of particles (i.e., the formation of a single particle from two colliding particles) is proportional to the GT product in secondary effluent flocculation. Special hardware has been developed to enhance the flocculation of sewage-like solids (9). Design and calculation methods have been developed so that the mixing intensities as measured by velocity gradient can be controlled in the laboratory (10) (11) and also reproduced in full scale equipment (12).

The application of this already developed mixing intensity technology to disinfection has been proposed by the writer (13).

The following will be a description of (a) the performance of the pilot units, (b) the preliminary design scheme, and (c) of a 92 cfs chamber designed according to this scheme.

Figure 2 shows the results of our disinfection studies to date on combined sewer overflow in an intensely mixed chlorine contact chamber. The kill is shown as the surviving fraction of the total coliform on a log scale. Note that almost 4 logs (99.99%) are obtained with 10 ppm dosage at GT of 5,000 (2 minutes at G = 40). The contact time-mixing intensity scale is dimensionless. It is based on the nominal contact time; that is, the volume of the chamber divided by the thruput rate and is not corrected for short circuiting. The value of 9,500, for example, is the product of the G = 40 sec^{-1} velocity gradient times 240 seconds (4 minutes) nominal contact time.

For comparison, the velocity gradient in the contact chamber of a local



STIRRED (2)
TUBE (2)
CORRUGATED BAFFLE
FRACTION OF SURVIVING ORGANISMS
GT
5mg/l
5mg/l
10mg/l
15 mg/l
10^{-0}
10^{-1}
10^{-2}
10^{-3}
10^{-4}
10^{-5}
10^{-6}

FIGURE 2

sewage plant was calculated from observed velocity and head loss and found to be about 6 sec^{-1}. The nominal residence time in this chamber was 1,800 seconds (30 minutes) and the GT product then was 10,000. It might be noted here that the nominal residence time is used although it has been shown (2) (14) that the true residence time is often considerably less due to short circuiting. Preliminary studies have indicated that the use of a true residence time would improve this scheme but this refinement has not been incorporated yet.

The design objective for our pilot chambers was to achieve a GT of 10,000. We arbitrarily selected 240 seconds (4 minutes) as the residence time T so that we needed a G of about 40 sec^{-1}. The velocity gradient is defined (G) as:

$$G = \left[\frac{\text{Energy Dissipation Rate/Volume}}{\text{Viscosity}}\right]^{1/2}$$

For open channel flow, it has been shown (12) that:

$$G = \frac{1730}{\sqrt{\text{viscosity-cp}}}\sqrt{\text{Velocity-fps} \times \text{Channel Slope ft/ft}} \qquad \text{(Eq. 1)}$$

The viscosity is known from the lowest temperature to be considered in the design; e.g., 1.4 centipoise at 45° F.

The velocity can be arbitrarily selected at some level between 0.25 and 1.5 ft/sec, or possibly higher. The volume of the chamber has already been determined by the selected nominal residence time so that now the velocity selection also fixes the path cross-sectional area and path length.